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THE STRENGTH OF WROUGHT IRON COLUMNS.

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READ SEPTEMBER 1ST, 1880.

The construction of the bridge over the Ohio River for the crossing of the Cincinnati Southern Railway in 1875, and the subsequent letting during the same year of the contracts for nineteen other iron bridges and viaducts for the same road, including the important structures over the Kentucky and Cumberland Rivers, were the occasion of my first experiments on the strength of wrought iron columns.

Gordon's and Rankine's empirical formulæ, deduced from Hodgkinson's experiments on solid bars of small sizes, and used almost exclusively by American engineers for the determination of the sectional areas of compression members, are as follows:

$$\text{Gordon's } \frac{P}{S} = \frac{f}{1 + \frac{\alpha l^2}{h^2}}$$

$$\text{Rankine's } \frac{P}{S} = \frac{f}{1 + \frac{\alpha' l^2}{r^2}}$$

P = Ultimate load in lbs., producing the crushing or bending of column.

S = Sectional area of column, in square inches.

f = Constant, supposed to be equal to the ultimate resistance per square inch of a short column whose length is equal to its diameter.

$$\alpha = \text{Constant} \left\{ \begin{array}{l} \text{For columns with flat bearings at ends} = \frac{1}{30000} \\ \text{For columns with flat bearing at one end} \\ \quad \text{and rounded at the other} \dots\dots\dots = \frac{2}{30000} \\ \text{For columns rounded at both ends} \dots\dots = \frac{4}{30000} \end{array} \right.$$

$$\alpha' = \text{Constant} \left\{ \begin{array}{l} \text{For columns with flat bearings at both ends} = \frac{1}{30000} \\ \text{For columns with flat bearing at one end} \\ \quad \text{and rounded at the other} \dots\dots\dots = \frac{2}{30000} \\ \text{For columns rounded at both ends} \dots\dots = \frac{4}{30000} \end{array} \right.$$

l = Length of column in inches.

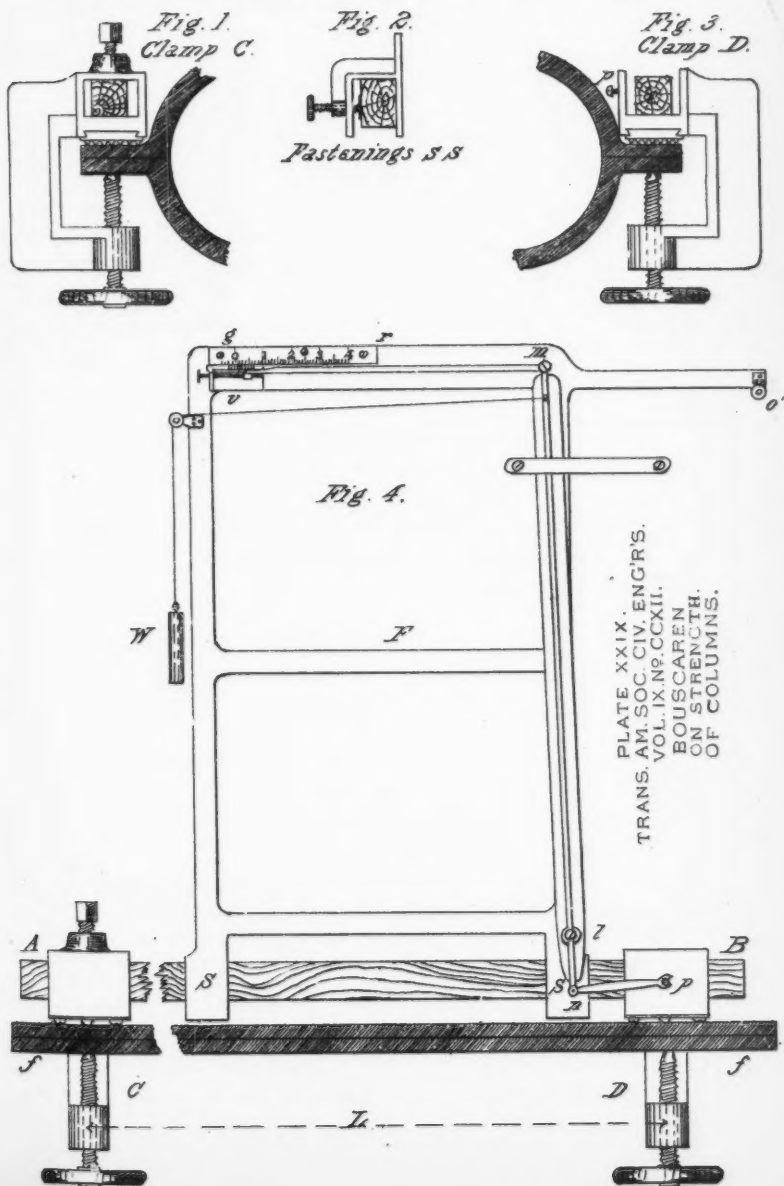
h = Diameter of column in the direction of its greatest deflection.

r = Radius of gyration of cross section of column in the direction of its greatest deflection.

Gordon's formula, the most popular of the two, seemed to be accepted without sufficient experimental evidence of its correctness, as applied to a great variety of shapes, almost every individual bridge builder having his preferred value for Constant " f ."

In preparing the specifications for the above named structures, I prescribed Gordon's formula, with the proviso that experiments would have to be made to determine the value of the constant " f " to be used for each kind of iron and shape of column proposed by the contractors.

*Apparatus to measure Compression and Extension of
Wrought-Iron Columns and Tie-Bars under strain.*





Thirty-four experiments were made during the year 1875 in accordance with these specifications, whereby columns and struts of various shapes and of the full size to be used in the structures, were crippled under pressure in a hydraulic press.

Tables numbers 1, 2, 3, 4, and 5, give the sizes and other characteristics of each of the columns tested, the results of each test and the corresponding calculated values of the constants for Gordon's and Rankine's formulæ.

In 1877, Experiments Nos. 35 and 36, Table No. 5, were made on the occasion of the construction of the bridge across the Tennessee River for the Cincinnati Southern Railway, to verify the assumption made in the calculations, that an open column composed of two channel bars latticed should be calculated by Rankine's formula with the same constant as for a closed column composed of channel bars and plates of the same kind of iron.

In 1878 and 1879, similar latticed posts having been adopted in the designs of the thirty-four iron structures contracted for by the Keystone Bridge Company, for the completion of the road, experiments No. 37 to 43, inclusive, given in Table No. 5, were made to test the result previously given by Experiments Nos. 35 and 36, and to regulate the spacing of the lattice bars and the thickness of metal in the channel bars.

In all these experiments the posts were placed horizontally in the press, without any intermediate guide or support, excepting a counter-balance in the centre, of about one-half their weight. The pressure, recorded by a mercurial gauge, was applied by increments of from 1000 to 2000 pounds per square inch of sectional area of column, and completely released between each application.

The vertical and lateral deflection and the compression of the column under each application of pressure and its permanent deformation after each release were carefully measured.

Figures 1 to 4, Plate XXIX, show the apparatus used for measuring the compression.

A brass frame F , bears a lever $m l n$, the long arm of which commands, by means of rod $m v$, a vernier, v , moving on a graduated inch scale $g r$. The small arm bears an articulated rod $n p$ forked at the loose end p . Two footings, $S S$, at the bottom of frame F , allow of its being fastened by means of clamp screws on to a wooden bar $A B$, made fast at one end

to flange ff of column by clamp C , and moving freely at the other end through clamp D as a guide. Clamp D is fastened to the flange of the column at such a distance from frame F as will allow the forked end of rod np to bear on a small pin p , borne by clamp D .

A small weight W , maintains the contact at p . The distance moved by point p being the compression of the length L of column between clamps C and D is multiplied ten times on scale gr , the vernier reading the $\frac{1}{1000}$ of an inch, the compression is observed to the nearest $\frac{1}{1000}$ of an inch, which for a length of twenty feet and a modulus of 24 000 000 corresponds to a pressure of fifty pounds per square inch.

After all parts had been brought to a good bearing by the first application of pressure (from five to eight thousand pounds per square inch) there was generally no permanent deformation observable until a pressure had been reached of about 15,000 pounds per square inch for the long columns, when a slight permanent set in deflection and compression would usually appear simultaneously and increase gradually until the crippling point was reached, when the column would yield faster than the pressure could be maintained by the pump.

The compression of the column during the period of perfect elasticity was carefully noted and used for the determination of the modulus. The modulus thus obtained is undoubtedly somewhat smaller than the modulus of the metal, a part of the observed compression being due to the bending of the column, but this error of observation being nearly the same for all columns of the same length, does not affect materially the comparison of the results. The same may be said of the influence of the neglected friction of the plunger, and of the vibration due to the close proximity of heavy working machinery in the shops where the experiments were made.

These experiments, although very incomplete, and made under somewhat unfavorable circumstances, have enabled me to accomplish substantially the object for which they were instituted, viz.: the proportioning of the compression members in the structures built by different contractors with the same margin of safety, and they have led me to the following conclusions. derived from a careful comparison of results obtained.

1st.—As between Rankine's and Gordon's formulæ, Rankine's is the more accurate of the two. This is shown by the results of the tests on the Phoenix and closed square posts (tables 3 and 4), where the parts being

thoroughly united together, the large factor due to defective construction and workmanship may be considered as eliminated in a great measure. These results compare as follows :

EXTREME VALUES OF CONSTANTS OBTAINED FOR PHOENIX POSTS WITH
FLAT ENDS.

	Rankine.	Gordon.
Largest value.....	49 400	57 500
Smallest value.....	41 100	43 700
Difference.....	8 300	13 800
	Less than 17 %.	More than 22 %.

EXTREME VALUES OF CONSTANTS OBTAINED FOR SQUARE CLOSED POSTS WITH
FLAT ENDS.

	Rankine.	Gordon.
Largest value.....	39 800	47 300
Smallest value.....	38 300	39 800
Difference.....	1 500	7 500
	Less than 4 %.	More than 15 %.

2d.—Iron of the highest modulus does not necessarily make the strongest column.

This conclusion, which is at variance with the opinion entertained by many engineers, is shown in a remarkable manner by the same experiments on the Phoenix and closed square posts, where not only do the values of constant " f " fail to follow any proportional relation to the corresponding moduli, but the greatest value of " f " obtained for the Phoenix posts actually corresponds to the smallest modulus. It is proper to state here that although the average moduli of the Phoenix and square posts were very nearly the same, the iron in the two kinds of posts differed widely as to other characteristics, the Phoenix iron being a very close-grained, fibrous and hard metal, whilst the iron of the square posts was comparatively soft and of a coarser texture, which explains the difference in the average values of " f " obtained for these two kinds of posts.

3d.—It is essential in built columns to thoroughly fasten together the several parts or segments that they may act in a solidary manner as one

continuous metallic body, and to guard not only against lateral flexure or buckling of each part, but also against longitudinal relative motion.

The importance of these conditions is demonstrated by the great variation in the results of tests of columns of the same length and shape, made of the same quality of iron, but differing in the fastening together of their several parts, as shown by experiments 3 and 8, and 25 and 31, Table No. 1, where the substitution of close riveting at the ends in place of cast shoes bolted on, made a difference of 22 % and 17 % respectively in the strength of the columns.

The anomalous results of tests Nos. 19 and 41, Tables Nos. 2 and 5, where the columns failed by flexure in the direction of the largest radius of gyration are also due, very probably, to the insufficiency of the riveting.

4th. A due regard to the economy of material demands that the thickness of the metal, and in built columns, the spacing of the rivets be so proportioned that the column shall fail by flexure, as a whole, before any local buckling or flexure shall take place.

This is well illustrated by experiments Nos. 37 to 42, inclusive, Table No. 5, instituted for the purpose of determining the largest spacing of rivets and the smallest thickness of metal allowable for a particular style and length of post. Columns Nos. 37 and 38, having both failed by the buckling of the metal in the channel, and the local flexure of the channel between rivets, pieces of channels were tested of the same shape and sectional area as those used in the posts, and of lengths equal to the distance between the lattice rivets; they failed in the same manner as the channels had failed in the long columns, but under somewhat higher pressure per square inch, confirming the first results.

Column No. 41, in all respects similar to Nos. 37 and 38, but with thickness of metal slightly increased and spacing of rivets reduced to 20 inches, was then tried and failed by the simultaneous flexion and buckling of the metal in the channel and the bending of the column bodily, showing that the column was properly proportioned, being of equal strength in all its parts.

Test No. 42 confirmed No. 41, and it was established that for that ratio of length to diameter the thickness of the metal should not be less than $\frac{1}{16}$ of the distance between supports transversely, which confirms Fairbairn's rule, and that the distance between rivets longitudinally should be such that the length of channel spanning it, considered as a

column, and calculated by Rankine's formula, shall give the same resistance per square inch of area as the column itself treated in the same manner with the same constant " f ."

5th. It may be concluded from experiments Nos. 13, 14, 16, 17, 21, 36 and 43, made with columns hinged on pins at both ends, that Rankine's formula with constant " α " assumed at double its value as applied to columns with flat ends, is practically correct, the variations in the values of " f " derived from these tests, being probably attributable, in a large measure, to the difference in the fitting of the pin, a "close fit" giving invariably higher results, the friction resisting, of course, the tendency to flexion.

The very large variation of the calculated value of " f " for Nos. 36 and 42 must, however, be ascribed to another cause, viz.: the now well established property of iron, that its limit of elasticity is increased after the metal has been once subjected to a strain beyond its limit of elasticity and then allowed to rest. This circumstance accidentally occurred for column No. 36, as explained in the marginal note in the table.

Tests No. 6 and 18, Tables Nos. 2 and 3, were also made on columns which had previously been strained beyond their elastic limit, and further confirm the same property.

Many important questions concerning wrought iron columns as used in our modern structures remain to be elucidated by direct experiments.

It would be useful to test :

1st. The effect of the eccentricity of the line of pressure to the axis of the column which occurs frequently through the unequal tension of tie-bars.

2d. The effect of a force acting transversely to the axis of the column. This occurs as a rule in all horizontal or inclined compression members, and in all posts riveted to floor beams.

3d. The applicability of the Rankine formula to the tiers of a long column guided at intervals, as is the case for the continuous posts of high trestles and piers held in line by the systems of horizontal and vertical bracing, the tiers being assimilable to neither flat nor pin-end posts.

4th. The influence on the strength of a long post, of the support afforded by a stiff but yielding web, as it occurs for the top flange of plate girders and the top chord of partly over-grade trusses.

In this line of investigation our large bridge contracting firms

should take the lead. They have at hand the facilities for conducting leisurely and methodically such experiments. The contractor and the engineer should join hands in an endeavor to reduce from *six* to *five* or even *four*, the irrepressible "factor of safety" invented to cover two large fields, the "incidental" and the "unknown," the limits of which should gradually contract under the light of well observed facts.








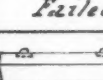

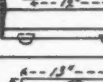


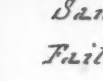


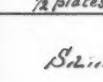



Experiments on Wrought Iron Columns

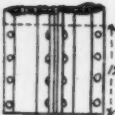

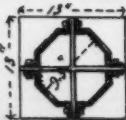
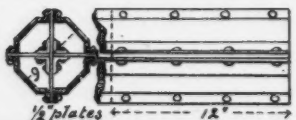
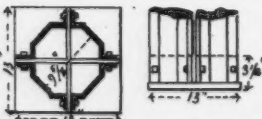
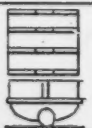
Table N^o1. Keystone Columns.



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N ^o	Date and Place of Testing		Place of Manufacture of Iron.	Kind	Cross Section at Centre	Length	Ratio of Length to Diameter	Distance from cen. to cen. of rivets - in. -	Sectional Area - sq. in. -	Square of Radius of Gyration. Unit=1 inch	Limit of Elasticity. - lbs. -	Resistance per sq. in. $\frac{P}{S}$ - lbs. -	Modulus of Elasticity.	$\frac{1}{2}$ " by Gordon Formula.	$\frac{1}{2}$ " by Rankine Formula.	Remarks.
1	April 19 th 1875.	Pittsburgh Penn.	Union Iron Mills Pittsburgh Penn.	Flat ends-closed.		0'-9"	1.1	5	14.25	—	—	45,000 (1) 51,500 (2)	—	—	—	Plated at both ends. No shoes. Rivetted through flanges Failed by buckling of iron in all segments (1) Signs of buckling (2) Crushing fast
7	April 27 th 1875	do	do	do		15'-0"	21.7	6	14.62	9.206	17,500	30,000	23,800,000	34,700	32,900	Plated at both ends. No shoes. Rivetted through flanges Failed by deflection
9	April 28 th 1875	do	do	do		15'-0"	20.3	6	23.67	7.833	19,000	32,000	24,700,000	36,400	35,700	Plated at both ends. No shoes. Rivetted through flanges With cross plates between flanges Failed by deflection
27	July 3 rd 1875	do	do	do		27'-0"	37.6	12	18.83	9.798	18,000	27,800	23,700,000	40,800	36,000	Plated at both ends. No shoes. Rivetted through flanges Iron soft and badly welded in rolling. Failed by deflection downward
24	July 2 nd 1875	do	do	Flat ends-open-straight		27'-0"	34.1	12	19.20	12.041	15,000	25,000	26,500,000	34,700	31,100	 Rivetted diametrically between flanges End fitted with diagonal plates and plated Failed by deflection sideways.
26	July 3 rd 1875	do	do	do		27'-0"	34.6	12	14.49	11.178	17,000	27,500	27,500,000	38,400	34,700	Rivetted diametrically between flanges Fitted with square castings bolted at ends Failed by deflection downward
30	Aug 5 th 1875	do	do	do		27'-0"	34.1	12	15.13	11.464	12,000	30,000	19,300,000	42,100	37,600	 Rivetted diametrically between flanges Ends plated. No shoes Failed by bending downward & sideways
2	April 19 th 1875	do	do	Flat ends open-swelled		5'-0"	6.5	15	14.25	11.044	—	33,600	—	34,100	33,900	 Rivetted diametrically between flanges Fitted with square castings bolted at ends Failed by buckling between rivets
																Same as N ^o 2

27	July 3 rd 1875	do	do	do		27'-0"	37.6	12	18.83	9.798	18,000	27,800	23,700,000	40,800	36,000	Iron soft and badly welded in rolling. Failed by deflection downward
24	July 2 nd 1875	do	do	Flat ends- open-straight		27'-0"	34.1	12	19.20	12.041	15,000	25,000	26,500,000	34,700	31,100	 Rivetted diametrically between flanges End fitted with diagonal plates and planed Failed by deflection sideways.
26	July 3 rd 1875	do	do	do		27'-0"	34.6	12	14.49	11.178	17,000	27,500	27,500,000	38,400	34,700	Rivetted diametrically between flanges Fitted with square castings bolted at ends Failed by deflection downward
30	Aug 5 th 1875	do	do	do		27'-0"	34.1	12	15.13	11.464	12,000	30,000	19,300,000	42,100	37,600	 Rivetted diametrically between flanges Ends planed. No shoes Failed by bending downward & sideways
2	April 19 th 1875	do	do	Flat ends open-swelled		5'-0"	6.5	15	14.25	11.044	—	33,600	—	34,100	33,900	 Rivetted diametrically between flanges Fitted with square castings bolted at ends Failed by buckling between rivets
3	April 19 th 1875	do	do	do		15'-0"	19.5	15	14.84	10.834	—	28,800	34,600,000	32,400	31,200	Same as N° 2 Failed by deflection
8	April 28 th 1875	do	do	do		15'-0"	20.0	15	14.80	10.353	15,000	36,900	29,600,000	41,800	40,100	 Rivetted diametrically, ends planed and rivetted with cross plates & angle irons. Failed by buckling between rivets
4	April 20 th 1875	do	do	do		27'-0"	35.2	15	12.96	10.883	16,700	24,100	—	34,100	30,600	Same as N° 2 diameter at ends 9.2" Failed by deflection
25	July 3 rd 1875	do	do	do		27'-0"	33.7	12	18.83	11.424	12,000	21,100	28,100,000	29,100	26,500	 Same as N° 2 and 26 Failed by deflection downward
31	Aug. 5 th 1875	do	do	do		27'-0"	34.1	12	15.13	11.464	16,000	25,400	23,600,000	36,100	31,900	Rivetted diametrically between flanges Ends as for N° 30. Failed by deflection upward & sideways
5	April 21 st 1875	do	do	Hinged ends open-swelled		27'-0"	35.1	15	13.12	10.945	15,000	22,000	29,500,000	40,100	33,700	 Rivetted diametrically. Square castings at ends fitting on a pin, the whole length of the latter bearing against an abutting casting. Diameter of column at ends = 9.22". Failed by deflection.

12	18.83	9.798	18,000	27,800	23,700,000	40,800	36,000	Iron soft and badly welded in rolling. Failed by deflection downward
12	19.20	12.041	15,000	25,000	26,500,000	34,700	31,100	 Rivetted diametrically between flanges End fitted with diagonal plates and planed Failed by deflection sideways.
12	14.49	11.178	17,000	27,500	27,500,000	38,400	34,700	Rivetted diametrically between flanges Fitted with square castings bolted at ends Failed by deflection downward
12	15.13	11.464	12,000	30,000	19,300,000	42,100	37,600	 Rivetted diametrically between flanges Ends planed. No shoes Failed by bending downward & sideways
15	14.25	11.044	—	33,600	—	34,100	33,900	 Rivetted diametrically between flanges Fitted with square castings bolted at ends Failed by buckling between rivets
15	14.84	10.834	—	28,800	34,600,000	32,400	31,200	Same as N° 2 Failed by deflection
15	14.80	10.353	15,000	36,900	29,600,000	41,800	40,100	 Rivetted diametrically, ends planed and rivetted with cross plates & angle irons. Failed by buckling between rivets
15	12.96	10.883	16,700	24,100	—	34,100	30,600	Same as N° 2 diameter at ends 9.2" Failed by deflection
12	18.83	11.424	12,000	21,100	28,100,000	29,100	26,500	 Same as N° 2 and 26 Failed by deflection downward
12	15.13	11.464	16,000	25,400	23,600,000	36,100	31,900	Rivetted diametrically between flanges Ends as for N° 30. Failed by deflection upward & sideways
15	13.12	10.945	15,000	22,000	29,500,000	40,100	33,700	 Rivetted diametrically. Square castings at ends fitting on a pin, the whole length of the latter bearing against an abutting casting. Diameter of column at ends = 9.22". Failed by deflection.

Experiments on Wrought Iron Columns.

Table N^o 2

American Bridge Co's Columns.



N ^o	Date and Place of Testing		Place of Manufacture of Iron	Kind	Cross-section at Centre	Length	Ratio of Length to Diameter	Dist. fr. cen. to cen. of rivets in.	Sectional Area sq. in.	Square of Radius of Gyration (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. lbs.	Modulus of Elasticity	"f" by Gordon Formula	"f" by Rankine Formula	Remarks.
12	June 2 nd 1875	Chicago Ills.	Cleveland Rolling Mills Cleveland O.	Flat ends.		20'-0"	25.3	8	20.10	8.653	15,000	—	—	—	—	Rivetting at ends. Ends planed to fit square. Uneven bearings; test disc of 19,000 lbs. per sq. in. h.
18	June 3 rd 1875	do	do	do		20'-0"	25.3	8	20.10	8.653	23,000	31,500	23,600,000	37,500	37,300	Same column as N ^o 12 tested & se. Failed by bending sideways.
19	June 3 rd 1875	do	do	do		27'-0"	34.1 [32.4]	8	20.10	13.510 [8.635]	24,000	27,800	32,900,000	38,600 [37,500]	33,700 [37,200]	Ends planed and rivetted as for N ^o 12. Failed by bending upward
15	June 3 rd 1875	do	do	do		30'-0"	45.0	8	14.97	5.388	13,000	23,700	26,000,000	39,700	39,500	Ends as for N ^o 12—Column balanced. Failed by bending sideways.
16	June 3 rd 1875	do	do	Hinged ends		20'-0"	30.0	8	12.50	5.479	15,000	26,700	28,900,000	42,700	42,300	Ends Hinged as for N ^o 13. Failed by bending sideways.
17	June 3 rd 1875	do	do	do		20'-0"	24.0	8	19.90	8.733	12,000	26,500	23,100,000	36,700	36,200	Ends Hinged as for N ^o 13. Failed by bending sideways.
13	June 2 nd 1875	do	do	do		26'-0"	29.0	8	25.05	18.215	12,000	24,000	30,400,000	37,500	31,100	Ends Hinged as for N ^o 13. Failed by bending sideways. plate 7/8 inch thick
14	June 3 rd 1875	do	do	do		26'-0"	31.2	8	20.72	8.733	14,000	22,000	26,000,000	36,300	35,600	Ends Hinged as for N ^o 13. Failed by bending sideways.



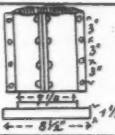




Table N^o 2 American Bridge Co's Columns.

Cross-section at Centre	Length	Ratio of Length to Diameter	Dist. fr. cen. to cen. of rivets in.	Sectional Area sq. in.	Square of Radius of Gyration (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. lbs.	Modulus of Elasticity	"f" by Gordon Formula	"f" by Rankine Formula	Remarks.
	20'-0"	25.3	8	20.10	8.653	15,000	—	—	—	—	Rivetting at ends. Ends planed to fit squarely on abutting castings. Uneven bearings; test discontinued after a pressure of 19,000 lbs. per sq. in. had been applied.
	20'-0"	25.3	8	20.10	8.653	23,000	31,500	23,600,000	37,500	37,300	Same column as N ^o 12 tested a second time. Failed by bending sideways.
	27'-0"	34.1 (32.4)	8	20.10	13.510 (8.635)	24,000	27,800	32,900,000	38,600 (37,500)	33,700 (37,200)	Ends planed and rivetted as for N ^o 12. Failed by bending upward. { The numbers between parenthesis apply to bending sideways.
	30'-0"	45.0	8	14.97	5.388	13,000	23,700	26,000,000	39,700	39,500	Ends as for N ^o 12—Column balanced at center with 500 lbs. Failed by bending sideways.
	20'-0"	30.0	8	12.50	5.479	15,000	26,700	28,900,000	42,700	42,300	Ends Hinged as for N ^o 13. Failed by bending sideways.
	20'-0"	24.0	8	19.90	8.733	12,000	26,500	23,100,000	36,700	36,200	Ends Hinged as for N ^o 13. Failed by bending sideways.
	26'-0"	29.0	8	25.05	18.215	12,000	24,000	30,400,000	37,500	31,100	Ends Hinged as shown. Failed by bending downward. plate 1/4" thick
	26'-0"	31.2	8	20.72	8.733	14,000	22,000	26,000,000	36,300	35,600	Ends Hinged as for N ^o 13. Failed by bending sideways.

Experiments on Wrought Iron Columns.

Table No 3 Phoenix Columns.



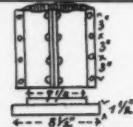



No	Date and Place of Testing		Place of manufacture of Iron	Kind	Cross-section at Centre	Length	Ratio of Length to Diameter	Dist. fr. cen. to cen. of rivets in.	Sectional Area sq. in.	Square of Radius of Gyration. (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. = $\frac{P}{A}$ lbs.	Modulus of Elasticity	f by Gordon Formula	f by Rankine Formula	Remarks
6	April 27 th 1875	Pittsburgh Penn.	Phoenixville Penn.	Flat ends		15'-0"	22.4	6	14.09	8.536	35,000	37,500	27,400,000	43,700	41,500	Planned at both ends. No shoes, previously. Failed by bending.
10	June 2 nd 1875	Chicago Ills.	Phoenixville Penn.	do		27'-0"	39.9	6	13.70	8.935	18,000	31,000	29,100,000	47,400	41,100	 Ends rivetted closely with square casting. Failed by bending.
28	Aug. 5 th 1875	Pittsburgh Penn.	Phoenixville Penn.	do		28'-0"	40.7	6	13.58	8.935	22,000	34,800	25,700,000	54,600	47,000	Ends rivetted closely and Balanced at center with 6 lbs. Failed by bending downwards.
29	Aug. 5 th 1875	do	Phoenixville Penn.	do		28'-0"	40.7	6	13.58	8.935	18,000	36,600	28,500,000	57,500	49,400	Ends planned and rivetted. Balanced at center with 6 lbs. Failed by deflection upwards.
11	June 2 nd 1875	Chicago Ills.	Phoenixville Penn.	Ends Rounded With large Radius		27'-0"	39.9	6	13.89	8.935	17,000	21,700	27,100,000	(1) 44,700 (2) 67,700	(1) 35,900 (2) 50,000	 Ends closely rivetted with spherical abutting. Failed by bending.

Experiments on Wrought Iron Columns.

Table N^o 3 Phoenix Columns.



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d	Cross-section at Centre	Length	Ratio of Length to Diameter	Dist. fr. cen. to cen. of rivets in.	Sectional Area sq. in.	Square of Radius of Gyration. (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. $\frac{P}{A}$ lbs.	Modulus of Elasticity	f' by Gordon Formula	f' by Rankine Formula	Remarks.
ds		15'-0"	22.4	6	14.09	8.536	35,000	37,500	27,400,000	43,700	41,500	Planed at both ends, No shoes, had been compressed previously. Failed by bending.
		27'-0"	39.9	6	13.70	8.935	18,000	31,000	29,100,000	47,400	41,100	 Ends rivetted closely and fitted with square castings as shown. Failed by bending.
		28'-0"	40.7	6	13.58	8.935	22,000	34,800	25,700,000	54,600	47,000	Ends rivetted closely and planed, Balanced at center with 650 lbs. Failed by bending downward.
		28'-0"	40.7	6	13.58	8.935	18,000	36,600	28,500,000	57,500	49,400	Ends planed and rivetted closely. Balanced at center with 650 lbs. Failed by deflection upward.
unded ge s		27'-0"	39.9	6	13.89	8.935	17,000	21,700	27,100,000	(1) 44,700 (2) 67,700	(1) 35,900 (2) 50,000	 Ends closely rivetted and fitted with spherical abutting castings. Failed by bending. (1) By formula for Hinged ends. (2) By formula for Round ends.

Experiments on Wrought Iron Columns.

Table N^o 4

Square Columns.



N ^o	Date and Place of Testing		Place of manufacture of Iron	Kind	Cross-section at Centre	Length	Ratio of Length to Diameter	Dist. fr. cen. to cen. of rivets in.	Sectional Area sq in.	Square of Radius of Gyration (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. $\frac{P}{A}$ lbs.	Modulus of Elasticity	"f" by Gordon Formula	"f" by Rankine Formula	Remarks
23	June 29 th 1875	Chicago Ills.	Pencoyd Iron Works near Philadelphia Penn.	Flat ends.		24'-0"	34.1	4	13.70	11.628	15,000	33,200	28,900,000	46,100	39,800	Column from the Baltimore. Ends planed. column broken. Failed by bending down.
22	do.	do	Ohio Falls Iron Works New Albany Indiana	do		26'-0"	41.6	4 1/2	13.60	9.347	16,000	30,000	27,800,000	47,300	38,700	Column from the Louisville. Closely rivetted, column broken. Failed by bending up.
32	Aug. 26 th 1875	Pittsburgh Penn.	Union Iron Mills, Pittsburgh Penn.	do		27'-0"	30.9	6	26.05	10.909	15,000	30,200	30,100,000	39,800	38,300	Column from the Keystone. Failed by bending sideways.
21	June 29 th 1875	Chicago Ills.	Ohio Falls Iron Works New Albany	Hinged ends		25'-9"	30.9	4 1/2	13.60	11.000	18,000	25,500	31,000,000	41,700	37,800	Column from the Louisville. (Table N ^o 2) Column broken. Failed by bending.

Experiments on Wrought Iron Columns.

Table N^o 4 Square Columns.




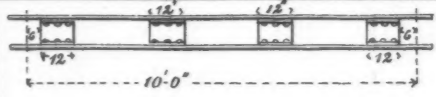

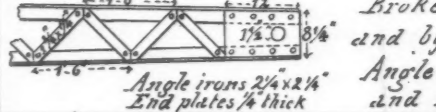

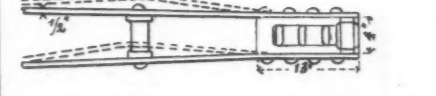

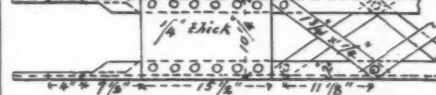
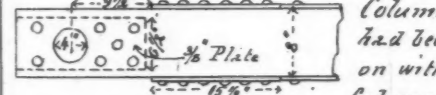

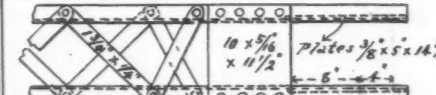
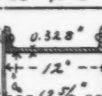

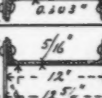
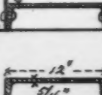

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Cross-section at Centre	Length	Ratio of Length to Diameter	Dist. fr. cen. to cen. of rivets in.	Sectional Area sq in.	Square of Radius of Gyration (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. $\frac{P}{A}$ lbs.	Modulus of Elasticity	"f" by Gordon Formula	"f" by Rankine Formula	Remarks
	24'-0"	34.1	4	13.70	11.628	15,000	33,200	28,900,000	46,100	39,800	Column from the Baltimore bridge Co. Twisted before test $\frac{1}{4}$ ". Ends pinned, column balanced at center with 300 lbs. Failed by bending downward, plates buckled.
	26'-0"	41.6	4 1/2	13.60	9.347	16,000	30,000	27,800,000	47,300	38,700	Column from the Louisville bridge Co. Ends pinned and closely rivetted, column balanced at center with 300 lbs. Failed by bending upward.
	27'-0"	30.9	6	26.05	10.909	15,000	30,200	30,100,000	39,800	38,300	Column from the Keystone Bridge Co. Ends pinned Failed by bending sideways.
	25'-9"	30.9	4 1/2	13.60	11.000	18,000	25,500	31,000,000	41,700	37,800	Column from the Louisville bridge Co. Castings as for N ^o 14 (Table N ^o 2) Column balanced with 300 lbs. at center Failed by bending sideways.

Experiments on Wrought Iron Columns.

Table N^o 5. Open Columns.

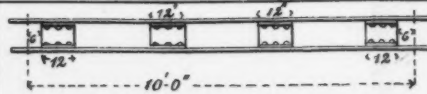
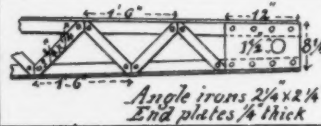
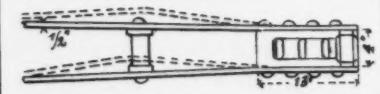
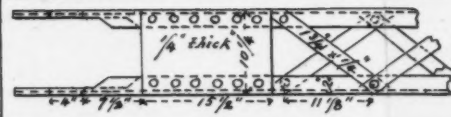
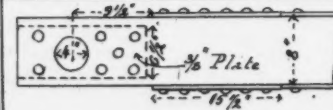
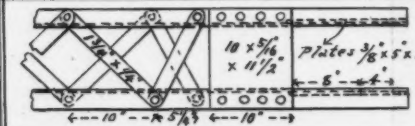
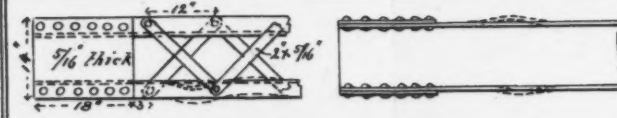
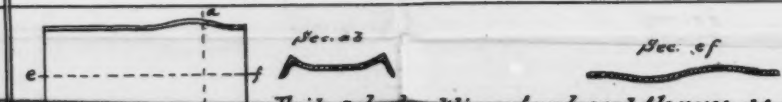
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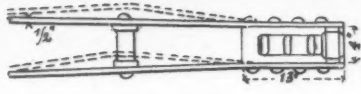

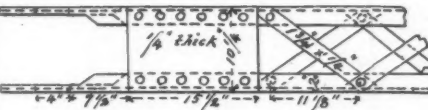
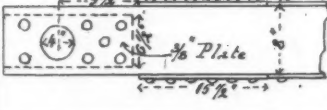

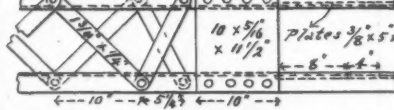
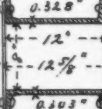
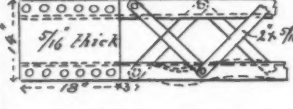
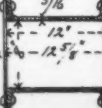
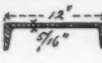

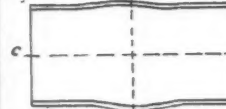
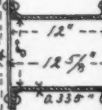

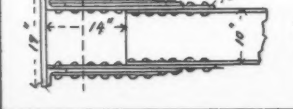
N ^o	Date and Place of Testing		Place of manufacture of iron	Kind	Cross-section at centre	Length	Ratio of length to Diam.	Distance from cen. to cen. of Rivets In.	Sectional Area. Sq. in.	Square of Radius of Gyration (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. lbs.	Modulus of Elasticity	f" by Gordon Formula	f" by Rankine Formula	Remarks.
20	June 3 rd 1875	Chicago Ills.	Cleveland Rolling Mills Cleveland, O.	Pin Ends		10'-0"	15.0	—	9.66	10.90	8,000	20,000	27,600,000	23,000	21,400	 Broke by deflection downward. Defective Construction.
33	Nov. 22 nd 1875	Pittsburgh Penn.	Cleveland Rolling Mills, Cleveland O.	do		28'-6 1/2"	34.24	18"	5.68	20.07	21,100	31700	32,400,000	56,500	42,000	 Broke by deflection upward and sideways and by buckling of angles between lattice. Angles fractured at one end near plates and near middle of flut. Angle irons 2 1/4 x 2 1/4 End plates 1/4 thick
34	Dec. 4 th 1875	Chicago Ills.	Ohio Falls Iron Works New Albany Indiana	Flat Ends		12'-3"	18.37	—	6.00	18.06	16,000	17,600	—	19,600	18,200	 Broke by deflection upward. Defective construction.
35	May 31 st 1877	Louisville Ky.	Union Iron Mills, Pittsburgh Pa.	Pin Ends		34'-0"	51.00	22 1/4"	7.48	8.73	8,800	20,053	—	—	—	 Column balanced by 435 lbs. gave way by pin crushing and splitting web of channel. Column not injured otherwise. No permanent set in deflection. Permanent set in compression 3 1/4 ft. = 0.121 in.
36	June 2 nd 1877	do	do	do	Same as N ^o 35	34'-0"	51.00	22 1/4"	7.48	8.73	21,600	23,128	—	63,100	47,600	 Column N ^o 35 tested again after crushed end had been cut off and thickening plates rivelled on with pin holes 34" from center to center. Column failed by deflection downward.
43	Feb. 18 th 1879	Pittsburgh Penn.	do	do		26'-6 1/2"	45.5	20"	6.50	5.95	12,000	18,000	—	42,800	35,049	 Counter balanced at centre with one half its weight. Failed by bending sideways at right angles with pins, without buckling of metal.
37	Dec. 23 rd 1878	do	do	Flat Ends		27'-5 1/8"	27.47	24"	12.078	19.98	—	29,600	—	38,000	34,094	 Webs buckled in both directions, in middle at one end of column column did not bend.
38	Dec. 30 th 1878	do	do	do		23'-0 1/8"	23.01	21"	13.476	20.69	23,000	32,300	—	38,000	35,626	Column balanced with 640 lbs. at center. Failed by buckling of web at both ends and at middle, same as N ^o 3 and by column deflecting downward simultaneously.
39	do	do	do	do		24"	8.53	—	6.6	0.7	—	35,400	—	36,200	35,264	 Failed by buckling of web and flanges as shown.

Experiments on Wrought Iron Columns.

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Table N^o 5. Open Columns.

Height length to diam.	Distance from cen. to cen. of Rivets In.	Sectional Area. Sq. in.	Square of Radius of Gyration (Unit=1 in.)	Limit of Elasticity lbs.	Resistance per sq. in. $\frac{P}{A}$ lbs.	Modulus of Elasticity	f" by Gordon Formula	f" by Rankine Formula	Remarks.
15.0	—	9.66	10.90	8,000	20,000	27,600,000	23,000	21,400	 Broke by deflection downward. Defective Construction.
4.24	18"	5.68	20.07	21,100	31,700	32,400,000	56,500	42,000	 Broke by deflection upward and sideways and by buckling of angles between lattices. Angles fractured at one end near plates and near middle of flut. Angle irons 2 1/4 x 2 1/4 End plates 1/4 thick
8.37	—	6.00	18.06	16,000	17,600	—	19,600	18,200	 Broke by deflection upward Defective construction.
1.00	22 1/4"	7.48	8.73	8,800	20,053	—	—	—	 Column balanced by 455 lbs. gave way by pin crushing and splitting web of channel. Column not injured otherwise. No permanent set in deflection. Permanent set in compression for 34 ft. = 0.121 in.
1.00	22 1/4"	7.48	8.73	21,600	23,128	—	63,100	47,600	 Column N ^o 35 tested again after crushed ends had been cut off and thickening plates rivelled on with pin holes 3/4" from center to center. Column failed by deflection downward.
5.5	20"	6.50	5.95	12,000	18,000	—	42,800	35,049	 Counter balanced at centre with one half its weight. Failed by bending sideways at right angles with pins, without buckling of metal.
7.47	24	12.078	19.98	—	29,600	—	38,000	34,094	 Webs buckled in both directions, in middle and at one end of column, column did not bend.
13.01	21	13.476	20.69	23,000	32,300	—	38,000	35,626	Column balanced with 640 lbs. at center. Failed by buckling of web at both ends and at middle, same as N ^o 37 and by column deflecting downward simultaneously.
1.53	—	6.6	0.7	—	35,400	—	36,200	35,264	 Sec. a 2 Sec. of

	1875	Tenn.	Cleveland O.															Angle irons 2 1/4 x 1 1/4 End plates 1/4 thick	Angles fractured at one end near plates and near middle of flut.
34	Dec. 4 th 1875	Chicago Ills.	Ohio Falls Iron Works New Albany Indiana	Flat Ends		12'-3"	18.37	—	6.00	18.06	16,000	17,600	—	19,600	18,200				Broke by deflection upward Defective construction.
35	May 31 st 1877	Louisville Ky.	Union Iron Mills, Pittsburgh Pa.	Pin Ends		34'-0"	51.00	22 1/4	7.48	8.73	8,800	20,053	—	—	—				Column balanced by 435 lbs. gave way by pin crushing and splitting web of channel. Column not injured otherwise. No permanent set in deflection. Permanent set in compression for 34 ft. = 0.121 in.
36	June 2 nd 1877	do	do	do	Same as N ^o 35	34'-0"	51.00	22 1/4	7.48	8.73	21,600	23,128	—	63,100	47,600				Column N ^o 35 tested again after Crushed ends had been cut off and thickening plates rivelled on with pin holes 3/4" from center to center. Column failed by deflection downward.
43	Feb. 18 th 1879	Pittsburgh Penn.	do	do		26'-6 1/2"	45.5	20"	6.50	5.95	12,000	18,000	—	42,800	35,049				Counter balanced at centre with one half its weight. Failed by bending sideways at right angles with pins, without buckling of metal.
37	Dec. 23 rd 1878	do	do	Flat Ends		27'-5 1/8"	27.47	24	12.078	19.98	—	29,600	—	38,000	34,094				Wells buckled in both directions, in middle and at one end of column, column did not bend.
38	Dec. 30 th 1878	do	do	do		23'-0 7/8"	23.01	21	13.476	20.69	23,000	32,300	—	38,000	35,626				Column balanced with 640 lbs. at center. Failed by buckling of web at both ends and at middle, same as N ^o 37 and by column deflecting downward simultaneously.
39	do	do	do	do		24'	8.53	—	6.6	0.7	—	35,400	—	36,200	35,264				Sec. ab Failed by buckling of web and flanges as shown.
40	do	do	do	do	Same as N ^o 39	19 5/8"	6.86	—	6.6	0.7	—	35,700	—	36,400	36,415				Sec. ab Sec. cd Failed by buckling of web and flanges as shown.
41	Feb. 18 th 1879	do	do	do		27'-6"	27.50	20	13.74	20.79	20,800	32,400	—	40,600	37,174				Column built the same as N ^o 37 with lattices spaced 20" instead of 24", balanced at center with one half its weight. Failed by buckling of web as for N ^o 37 and by bending downward and sideway simultaneously.
42	do	do	do	do		27'-6"	33.0 25.4	18	11.052	13.43 21.26	21,000	32,300	—	44,100 39,300	39,632 36,949				Balanced by one half its weight. Failed by deflecting upward, without buckling of metal.

									<div> <div> <p>Angle irons 2 1/4 x 2 1/4 End plates 1/4 thick</p> </div> <div> <p>Angles fractured at one end near plates and near middle of flut.</p> </div> </div>
37	—	6.00	18.06	16,000	17,600	—	19,600	18,200	<div> <p>Broke by deflection upward Defective construction.</p> </div>
0	22 1/4"	7.48	8.73	8,800	20,053	—	—	—	<div> <p>Column balanced by 435 lbs. gave way by pin crushing and splitting web of channel. Column not injured otherwise. No permanent set in deflection. Permanent set in compression for 34 ft. = 0.121 in.</p> </div>
00	22 1/4"	7.48	8.73	21,600	23,128	—	63,100	47,600	<div> <p>Column N° 35 tested again after crushed ends had been cut off and thickening plates rivelled on with pin holes 3 1/2" from center to center. Column failed by deflection downward.</p> </div>
5	20"	6.50	5.95	12,000	18,000	—	42,800	35,049	<div> <p>Counter balanced at centre with one half its weight. Failed by bending sideways at right angles with pins, without buckling of metal.</p> </div>
47	24	12.078	19.98	—	29,600	—	38,000	34,094	<div> <p>Welds buckled in both directions, in middle and at one end of column, column did not bend.</p> </div>
01	21	13.476	20.69	23,000	32,300	—	38,000	35,626	<p>Column balanced with 640 lbs. at center. Failed by buckling of web at both ends and at middle, same as N° 37 and by column deflecting downward simultaneously.</p>
3	—	6.6	0.7	—	35,400	—	36,200	35,264	<div> <p>Sec. ab</p> <p>Sec. ef</p> <p>Failed by buckling of web and flanges as shown.</p> </div>
6	—	6.6	0.7	—	35,700	—	36,400	36,415	<div> <p>Sec. ab</p> <p>Sec. cd</p> <p>Failed by buckling of web and flanges as shown.</p> </div>
50	20	13.74	20.79	20,800	32,400	—	40,600	37,174	<p>Column built the same as N° 37 with lattices spaced 20" instead of 24", balanced at center with one half its weight. Failed by buckling of web as for N° 37 and by bending downward and sideway simultaneously.</p>
04	18	11.052	+13.43 21.26	21,000	32,300	—	+44,100 39,300	+39,632 36,949	<div> <p>Balanced by one half its weight. Failed by deflecting upward, without buckling of metal.</p> </div>

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CCXIII.

(Vol. IX.—December, 1880.)

THE IMPROVEMENT OF THE HARBOR OF QUEBEC.

By J. VINCENT BROWNE, C. E.

READ SEPTEMBER 1ST, 1880.

Fortunately for Quebec, two great undertakings immediately connected with its future prospects, are now so far advanced as to demonstrate to some extent the purpose for which they were intended.

The North Shore Railroad is complete, and doing a fair amount of business, transporting lumber of all dimensions to this place. This railway and the improvements of the harbor will form an important factor in the prosperity of Quebec.

The great width of the St. Lawrence river at Quebec and its almost perpendicular rocky fore-shore, the unusual depth of water in places close in shore, with strong currents and the great rise and fall of the tide, render it necessary that some sort of harbor of refuge should be provided, so that vessels under detention or late in arriving can winter here in safety. Access to deep water at all stages of the tide must, also

be provided for the carrying trade opened by the North Shore Railroad.

The immediate vicinity of the mouth of the St. Charles river was selected for commencement of the present Harbor Improvements, which are now rapidly approaching completion.

Knipple & Morris, of London, England, are the designing engineers, but the works are under the immediate supervision of Woodford Pilkenton, also of London, England, all of whom are members of the Institution of Civil Engineers. The contract for these works was awarded to Simon Peters, of Quebec, Edward Moore, of Portland, Maine, and Augustus R. Wright of New York City. The contract holds the contractors responsible for the work and its maintenance for a period of twelve months after its final completion. It also requires the contractors to furnish a competent engineer to take charge and generally superintend the entire work.

Work was commenced in the fall of 1877, but little was then accomplished. During the succeeding seasons of 1878-9, rapid progress has been made. The work at present in progress of construction could form a centre embankment for a double wet dock, as well as a double tidal harbor, but one of each is all that is now contemplated. See Plate XXX.

The present embankment forms the northern wall, and the fore-shore the southern wall of the wet dock and tidal basin; the west wall of the wet dock is formed by Gas House Wharf, and the east wall by the cross-wall, which latter forms the west wall of the tidal harbor.

This embankment is named, "The Princess Louise Embankment." It has a length from east to west of 3 500 feet, a height above low water, of 24 feet, and extends from Gas House Wharf to, and forms a junction with what has been for some years an isolated mole or break-water, which stands in 50 feet of water at low tide, and is known as Ballast Wharf.

The south wall of the Louise Embankment, which is the north wall of the tidal harbor and wet dock, is nearly complete and is to be for its entire length a cut stone superstructure resting on a foundation of timber cribwork and concrete filling.

The north wall of the embankment is formed by a continuous (entremese filling) cribwork, with counterforts at intervals of every 20 feet, extending into the embankment 22 feet.

QUEBEC HARBOUR IMPROVEMENTS.

RIVER ST CHARLES

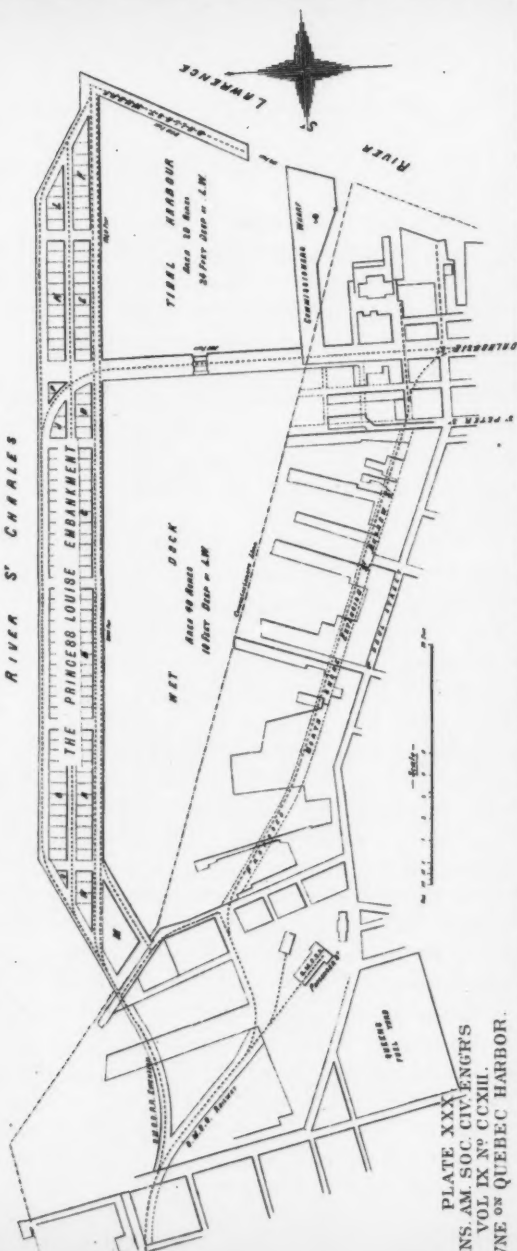
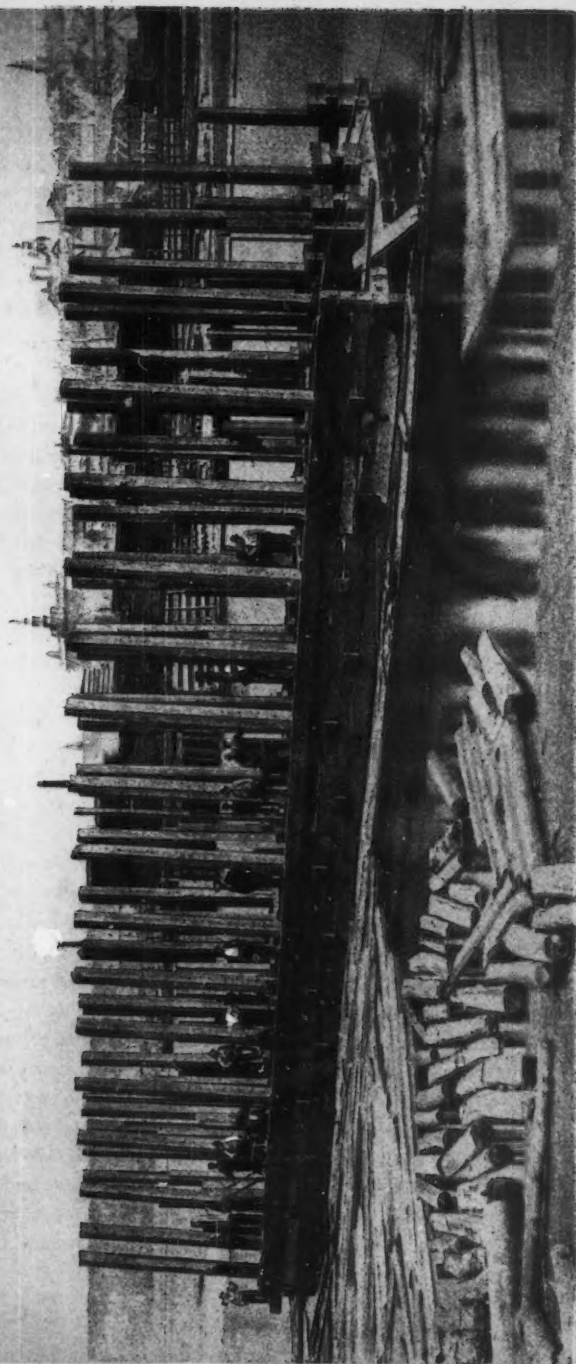
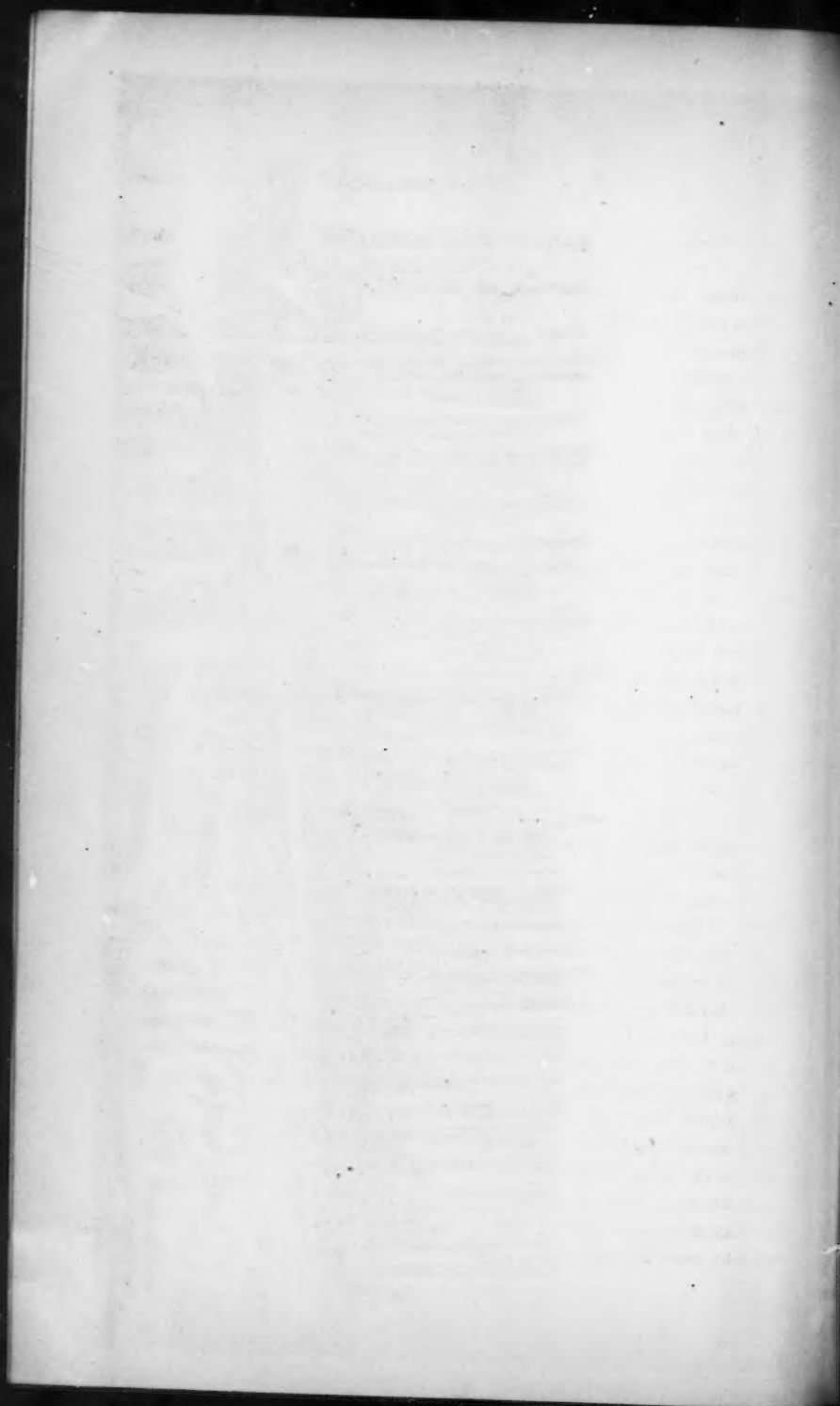


PLATE XXX
TRANS. AM. SOC. CIV. ENGRS
VOL. IX, No. CCXIII.
BROWNE ON QUEBEC HARBOR.



PLATE XXXI.
TRANS. AM. SOC. CIV. ENGR'S
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BROWNE ON QUEBEC HARBOR.





On final completion of this embankment there will be added to the length of the wharf accommodation of this port upwards of one and a half miles. The entire water area inclosed is sixty acres.

The works have already demonstrated their value as a protection to shipping, as not a dollars worth of damage has been sustained by the vessels within them last winter, although the ice was heavy. All the crowding and shoving that has taken place in former years, is now done away with.

The value of floating property wintered within the limits last season was upwards of two hundred and fifty thousand dollars.

There is now under consideration a steam ferry, for the purpose of transferring cars from the terminus of the North Shore Railroad, which is at the east extremity of the Louise Embankment to the opposite shore, or the Point Levis side of the St. Lawrence river, thereby giving through connection from the west to Halifax, Portland, Boston and New York.

For the foundations of the north wall of the wet dock and tidal basin a preliminary channel was dredged 150 feet in width, for 1 240 feet in length, to a uniform depth of 24 feet at low water; the remaining 2 310 feet, which falls inside the wet dock, was dredged to a depth of 10 feet at low water. No serious impediment was encountered, except the great rise and fall of the tide, which averages 18 feet, the spring tides often running as high as 22 and 23 feet. The work of dredging was pushed night and day.

The foundation for the quay wall of the tidal basin, is formed by a series of timber cribworks in lengths of 120 feet each, of the name and style known as entremese filling. (See Plate XXXI for front view, Plate XXXII for back view, Plate XXXIII top view on section of crib, and Plate XXXIV end elevation of crib). The dimensions of these cribs are, length, 120 feet, width at base 33 feet, width on top 23 feet, and in height 27 feet, batter of face one-half inch to the foot. Plate XXXIII shows the pockets formed in the front of the cribs by a series of planking, which form the rear retaining wall for the concrete facing of foundation. Each second crib that is sunk is provided with a bulkhead, at its west end, or at the end where the next crib sunk will join. This bulkhead prevents any current in the pockets during the rise and fall of the tide, and also gives comparatively still water in which to deposit the concrete, thereby guarding against any possible wash of the concrete after being

deposited. The back of the cribs immediately in the rear of the pockets (see Plate XXXIV) is filled with stone and clay, in nearly equal portions, allowing it to take its natural slope. The filling taken with the concrete face forms a most substantial foundation. The amount of concrete in each length of 120 feet is 1 650 cubic yards (equal to 2 475 tons), and of clay and stone filling 2 723 tons, so that the dead weight actually placed with care in each crib is 5 198 tons.

The dredged material is then filled in over this clay and stone filling to a level of one foot below the concrete.

This style of foundation is 1 240 feet long, and all falls within the tidal basin. The remaining 2 310 feet of foundation, or that portion which falls within the wet dock (see Plate XXXV) is formed of skeleton cribs, with plank forming the back of pockets, the same as in the tidal harbor, while the face of the cribs is formed by sheet piling. These cribs are brought up to the piles and then filled with concrete, stone and clay filling and backing.

These foundation cribs, in all instances, rest on a series of stub piles driven to a uniform level by means of a follower, and in no instance are these piles allowed to project above the surface of the bed of the channel over 12 inches. Four submarine divers have been employed, as with the great depth of water it would be impossible to secure a level bottom and remove various obstructions without their aid.

The mode of placing the concrete within the crib foundations is by a skip, or box holding 1 cubic yard (see Plate XXXVI). This skip is an invention of Edward Moore, and is covered by letters patent. It is made of boiler plate iron, is 3 feet square and 5 feet in height, and is provided with double doors as a cover. The lower part of this skip also has double doors, about two feet from the bottom, held in position by a strong spring latch, to which is attached a line, so as to spring the latch at will. When the skip is lowered to the bottom the latch is sprung, and the concrete deposited and retained within its walls. The skip is then raised from it, leaving the concrete in block, with the least possible wash. This mode is employed for all under-water work.

The mixing of concrete is done by a machine similar to the one used on the jetty works at the mouth of the Mississippi river, but instead of its being a fixture it is placed on a scow 75 feet long, 30 feet wide and 7 feet sides, drawing about 3 feet of water. In connection with the mixer is a swinging crane, with a traveling truck on its top arm. The proper

PLATE XXXII.
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BROWNE at QUEBEC HARBOR.



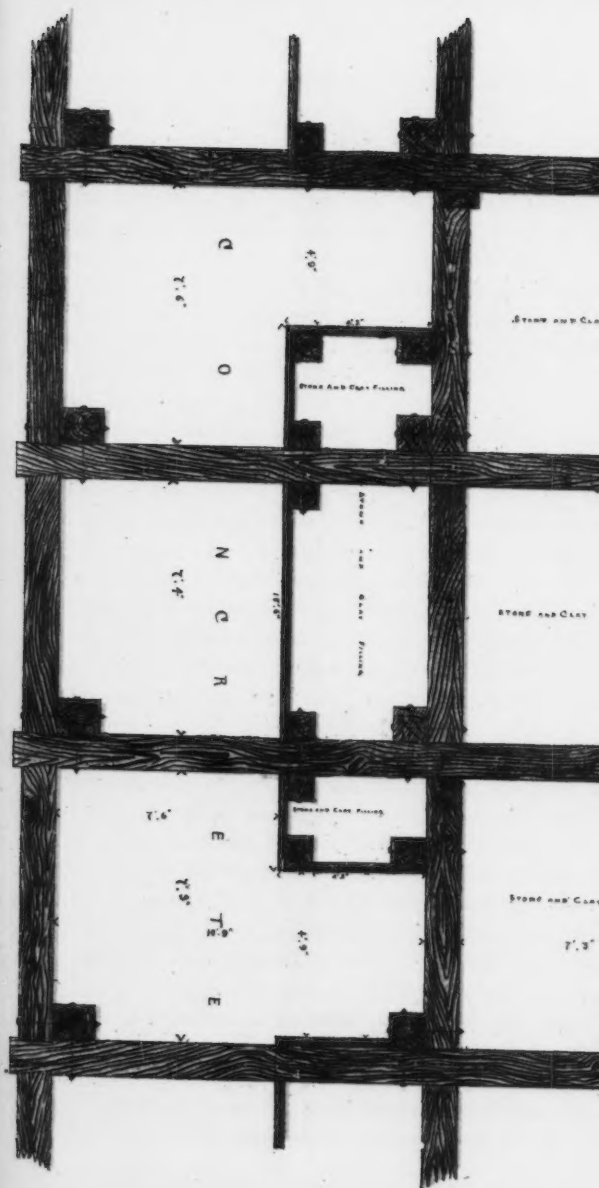
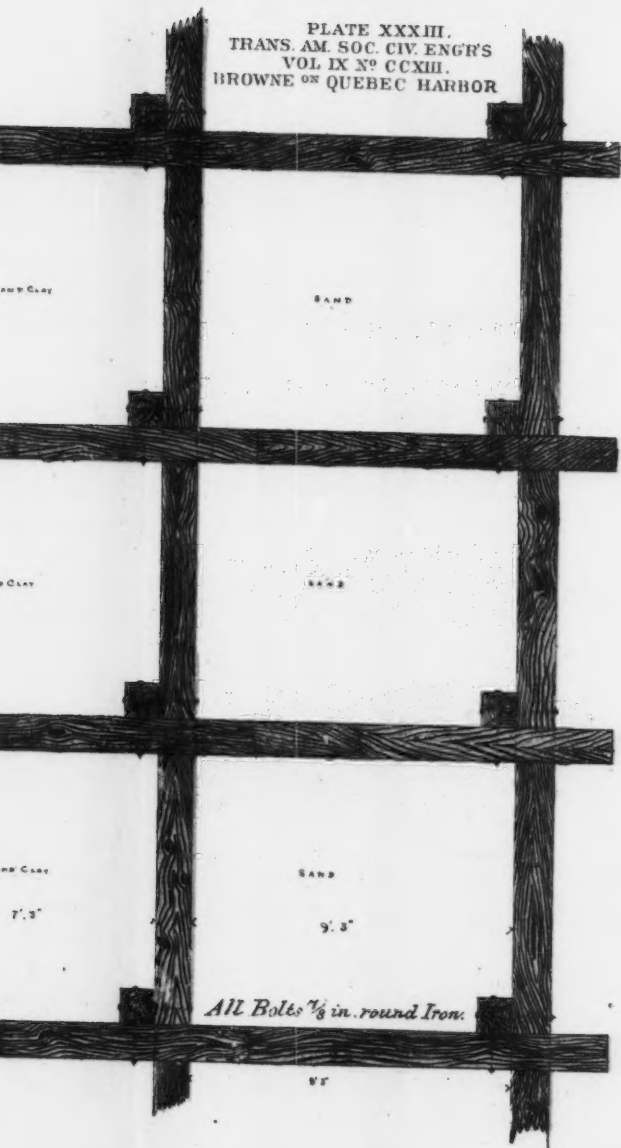


PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGR'S
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BROWNE ON QUEBEC HARBOR



portions of cement, sand and stone are placed in alternate layers, in an iron tub, and the requisite amount of water is deposited equally over the top of these, then it is hoisted up, and by means of a traveling truck, is run immediately over the hopper, which is directly over the mixer. The contents of the tub are dumped into this hopper, from which it passes into the mixer. Here it receives from 6 to 8 revolutions, when it is dumped into a car immediately underneath, the car is run out under the crane which handles the skip from the car. The concrete is placed in the skip, the crane swung around, and the skip run either in or out, at will. When in position it is lowered, and the concrete deposited in its place. By this appliance a uniform deposit is made, the mixer being hauled ahead or dropped back, at will, for the entire length of the work, thereby securing as nearly as possible uniform courses of concrete for the whole.

Owing to the great rise and fall of the tide, and to enable the work to continue at high water, or after the crib work is covered, traverses are secured to all angles of the rear planking, this forming a perfect profile of rear of the pockets, at the highest stage of the tide.

During the season of 1879, that is, from July until the 1st of November, there were placed in position by the mixer alone upwards of 9 000 cubic yards of concrete, and by hand 5 000 cubic yards. That placed by hand formed the backing of the cut stone superstructure, this requiring expert workmen.

The manner of bringing up the stone wall, and concrete backing, is as follows: A course of stone is first laid, which forms the front retaining wall. Plank, following the line of the planking in crib foundations, is then brought up for a back retaining wall, and the concrete placed in between the masonry and the planking. Great care is taken that at least 2 inches of fine material shall be secured between all the large stone placed in the concrete, as well as between them and the rear planking and the masonry. This concrete is finished off with each course of masonry laid, and large angular stones are left projecting from 4 to 12 inches above the concrete backing, and embedded at least 3 inches into it, this forming a perfect bond when the next course of concrete follows. As fast as the wall is carried up the filling in with dredged material is carried on, backing up the wall to within a reasonable distance from the top, so that no sand or silt may deposit itself upon the concrete during high water.

As the material dredged goes to form the embankment between the north and south walls, and as dump scows could only be used at certain stages of the tide, and then only to a limited extent, some rapid and economical means for conveying this material had to be employed. The difficulty was overcome by Messrs. Edward Moore and Augustus R. Wright, who jointly planned, constructed and patented a revolving derrick (see Plate XXXVII) with one arm on either side, directly opposite each other, each 110 feet long. This works in connection with a dipper dredge parallel to it, and 110 feet distant. From each arm of the derrick is suspended a tub of boiler plate iron of a capacity of 3 cubic yards. These tubs are swung alternately to the float on which they are landed at the dredge's side, the dredge raises its dipper, filled capacity, 3 cubic yards, swings it around, and dumps its contents into the tub.

The tub is then hoisted, the derrick swung around, and the contents of the tub dumped 210 feet from where it was dredged. And so the operation goes on day and night, moving about 1 500 yards per day. At first the derrick was carried on blocking, but it now rests on a pontoon, and is pulled ahead or dropped back at will.

Another portion of the material is moved by a clam-shell dredge, which travels on top of the northern wall of the embankment on a railroad track. This is employed in connection with a clam-shell dredge in the tidal basin.

The latter deposits its dredged material into scows which are run around outside and along the face of the northern wall, or cribwork, and dumped. Here the dredge on the embankment picks it up and deposits it inside the wall, thus forming the embankment.

The extent of dredging is about 1 000 000 cubic yards, and consists (see Plate XXX) of dredging the tidal basin to a uniform depth of 24 feet at low water, and the wet dock to a uniform depth of 10 feet at low water, thus securing at all times 24 feet of water in the tidal harbor, and 10 feet in the wet dock, where formerly it was all dry land at low water.

At high tide there will be secured 37 feet of water in the tidal harbor, and when the gates are closed in the cross wall there can be maintained at all times 27 feet of water in the wet dock.

The plant used on the works has been 1 dipper dredge, capacity of bucket, 3 cubic yards; 1 revolving derrick same capacity; 1 clam-shell dredge, capacity of bucket, 2 cubic yards; 1 clam-shell hoister same capacity; 8 dump scows, capacity, 100 cubic yards each; 2 steam tugs; 4

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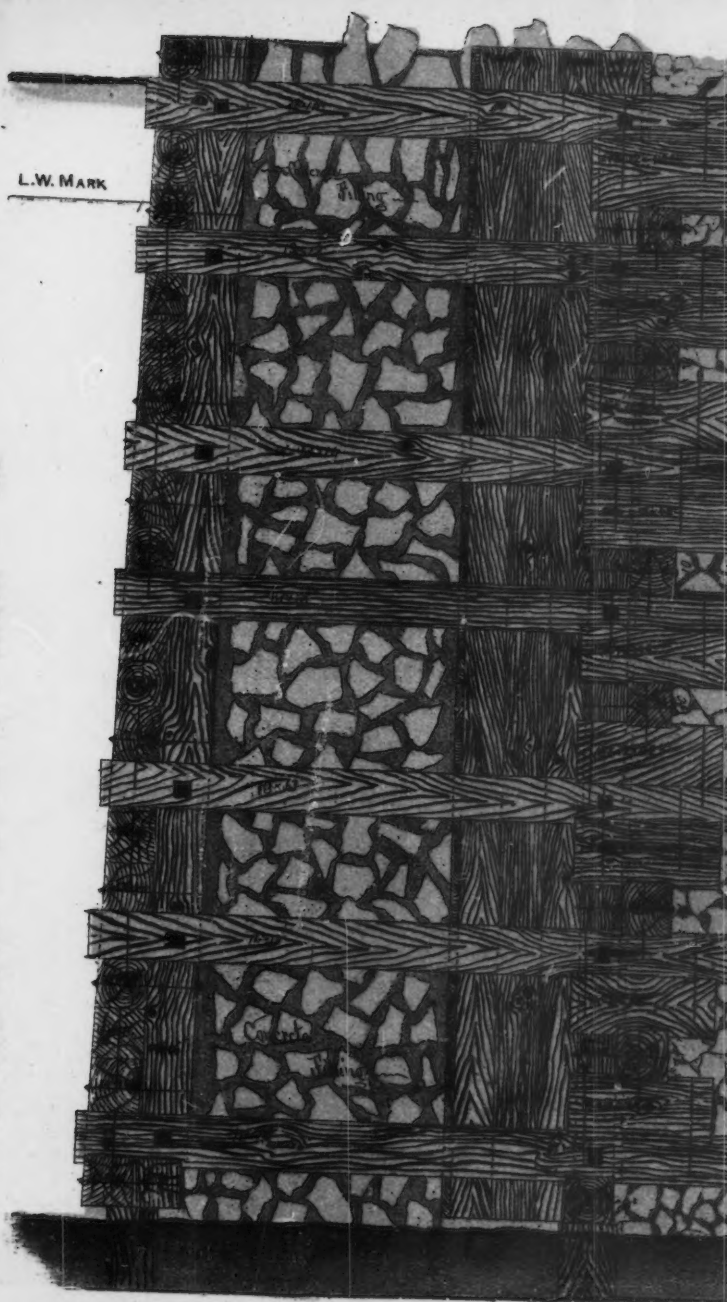
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L.W. MARK



FORMER SURFACE OF

LOW WATER MAR



OF GROUND AT LOW WATER

MARK

PLATE XXXIV.
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BROWNE ON QUÉBEC HARBOR.

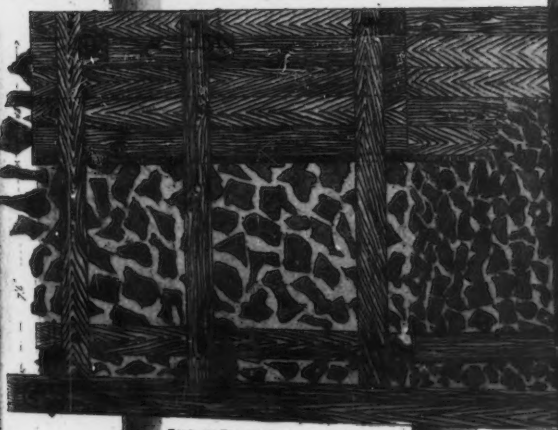


PLATE XXXV.
 TRANS. AM. SOC. CIV. ENGR'S
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 BROWNE ON QUEBEC HARBOR.

CRIB WORK FONDATION
 OF WET DOCK.

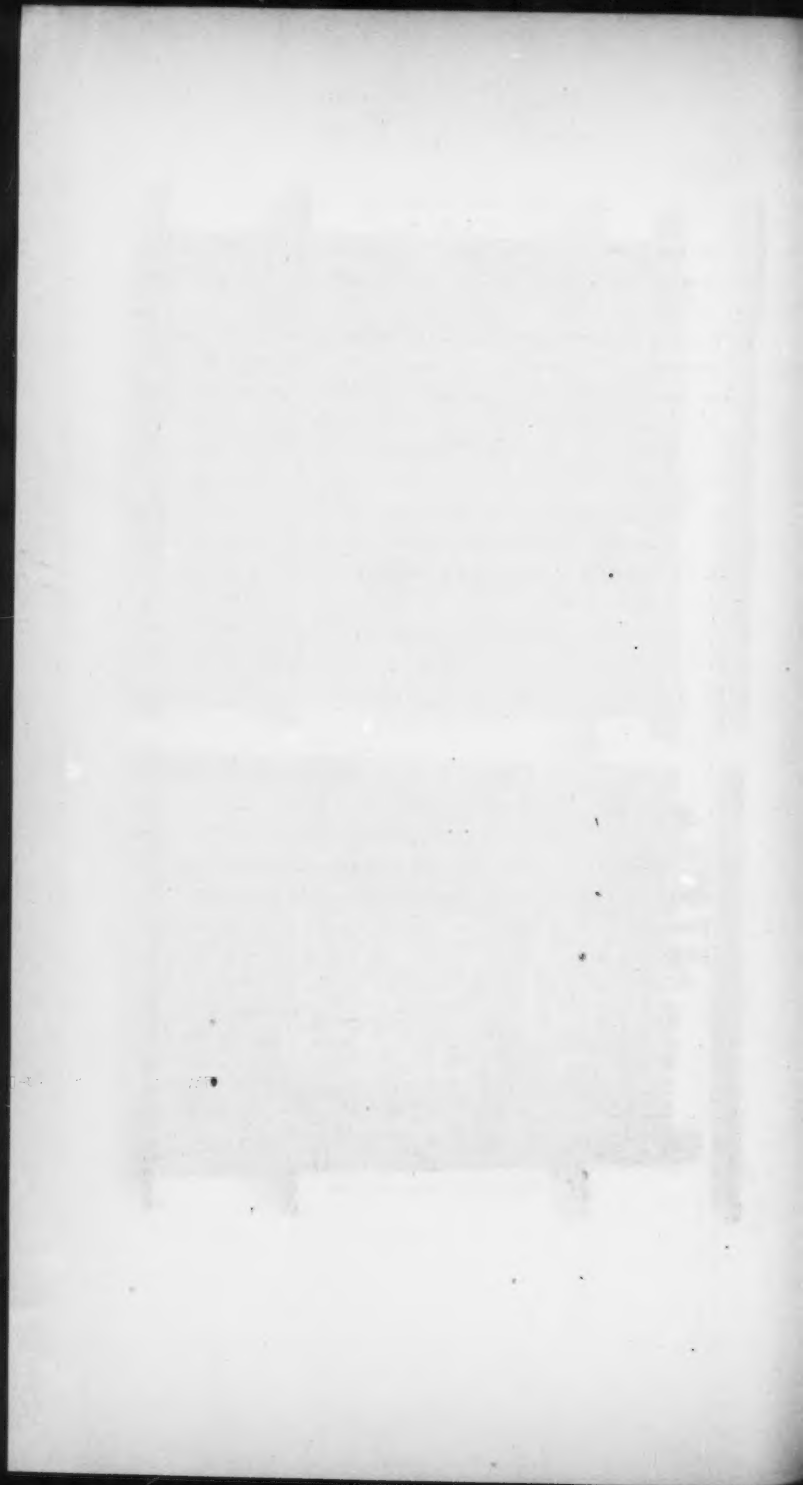


FRONT ELEVATION.



END ELEVATION AND SECTION.

Original Surface



deck scows; small steam hoister; steam concrete mixer; 3 pile drivers; 1 large barge (or cement store) beside numerous smaller craft for transporting supplies.

The concrete employed on these works consists of two kinds, eight to one and four to one. The former is composed of two barrels of clean, sharp sand, two of McAdam stone to pass a three-inch ring, four of large angular stone, and one of Portland Cement. The four to one concrete is composed of two barrels of clean, sharp sand, two of McAdam stone to pass a three-inch ring, and one of Portland Cement.

The cement used in these works is English Portland, manufactured by Gibbs & Co., London. Over 14 000 barrels have been used, all of which is subjected to a rigid test of its tensile strain, viz., 750 lbs. to a section of $1\frac{1}{2}$ inches square.

One feature should be particularly noticed, which is that cement transported on steamers will not show such good results as that brought by sailing vessels.

In one instance, when a cargo by steamer did not come up to the standard, although tested several times, I caused it to be aerated for one or two days, keeping a man turning it over gently. The result obtained after this process was satisfactory, being above the required standard. Many experiments also prove that the smallest amount of water that can be used and yet make a good stiff composition, without sweating or weeping, gives the best results, as is shown by the following table :

No. of Bbl.	Oz. of Neat Cement	Oz. of Water.	No. of Days in Water.	Tensile Strain $1\frac{1}{2}$ in. sq	Temperature.			REMARKS.
					Air.	Water.	Cement.	
573	25	$5\frac{1}{2}$	7	840	57	54	58	
573	25	4	7	1060	62	58	60	Minimum of water.
572	25	$5\frac{1}{2}$	7	770	58	54	58	
572	25	$5\frac{1}{2}$	7	300	58	54	58	
572	25	4	7	980	62	61	62	Minimum of water.
572	25	4	7	950	62	61	62	Minimum of water.

It will be seen that by the use of the minimum amount of water, a

considerable gain in strength is obtained. For testing tensile strength a lever-balance machine is used, manufactured by Patrick Adie, London, England.

All of our briquettes have an area of $1\frac{1}{2}$ square inches in the breaking section, but are sufficiently enlarged at the ends for the application of clamps.

CONDENSED TABLE OF CEMENT TESTS.

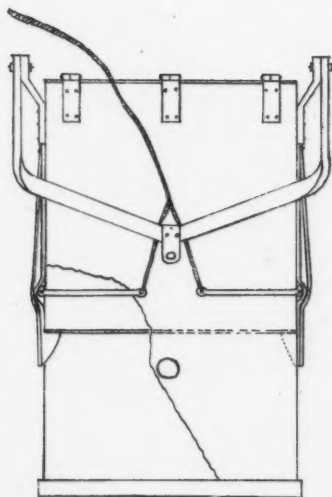
Date. 1879.	The number of briquettes from which average is taken.	Tensile Strain.		Percentage pass- ing sieve 250 meshes per sq. inch.	Weight in pounds of Imperial struck Bushel.	Average weight of bols. in lbs.	Number of ounces of neat cement to each Briquette.	Average Temperature.			Average ounces of Water in each Briquette.
		7 Days in Water.	30 Days in Water.					Air.	Water.	Cement	
June.	164	465.6	82	116½	473	25	56.7	54.7	56.2	5.33
June.	76	525.2	"	"	"	"	56.7	54.7	56.2	5.33
July.	194	528.94	608.32	"	115½	"	"	60.8	60.6	60.9	4.69
July.	94	608.32	"	"	"	"	60.8	60.6	60.9	4.69
Aug.	164	573.84	"	"	"	"	61.0	58.7	62.0	4.84
Aug.	82	646.6	"	"	"	"	61.0	58.7	62.0	4.84
Sept.	156	595.2	"	115½	"	"	56.2	54.6	56.2	4.70
Sept.	31	651.2	"	"	"	"	56.2	54.6	56.2	4.70

By referring to Plates XXXVIII and XXXIX some idea may be had of the difficulty experienced in maintaining work in course of construction in so severe a climate.

Plate XXXVIII shows a portion of the wall completed above reach of high water, to which is attached four feet in thickness of solid ice. This did not leave the wall until May 1st.

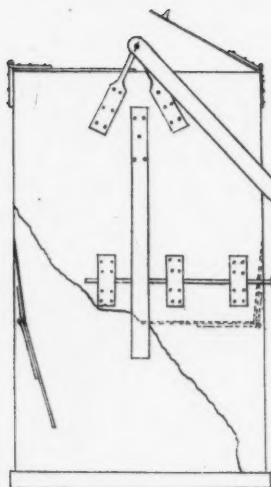
Plate XXXIX shows the immense body of ice resting upon the foundation of the wet dock wall, which is a continuance of the tidal basin wall shown in plate No. 10, again demonstrating the great difficulty that it has been to maintain the foundations through a winter season of six or seven months.

N^o 8.



Side view, showing hopper -

Concrete - Pump -



Side view, showing Catch & Cam -



Side view of Latch with cross-section of Cam -

PLATE XXXVI.
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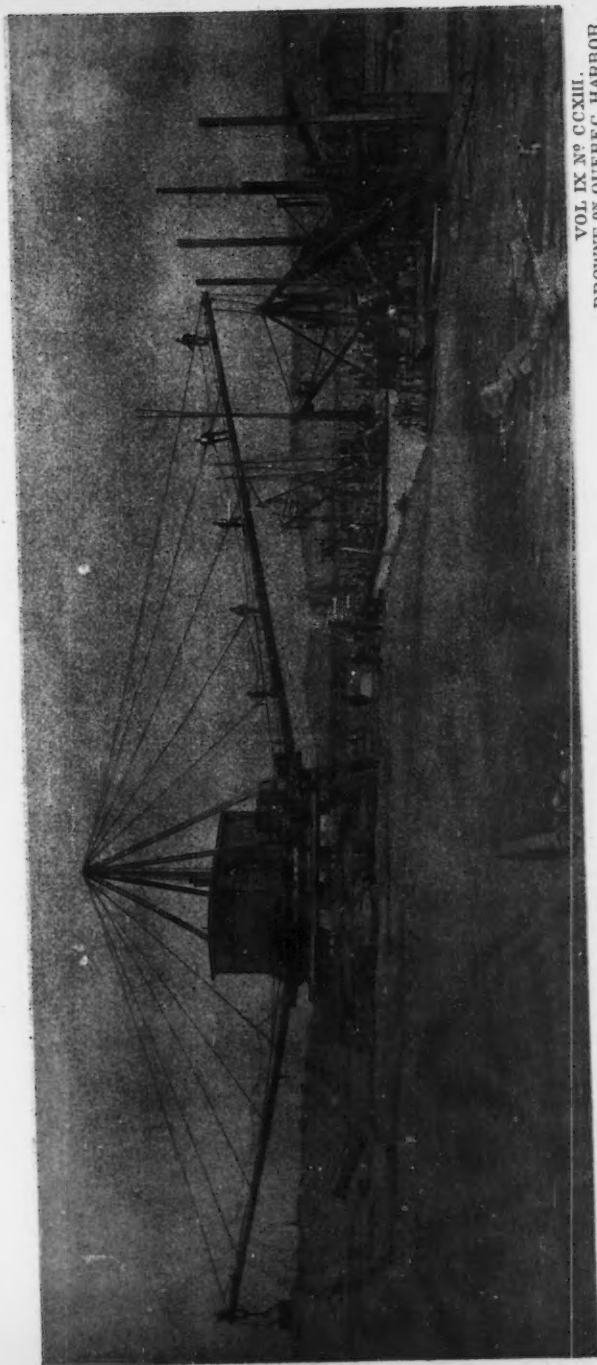


PLATE XXXVII.
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TRANS-ATLANTIC SHIPS
VOL IX 1906
BROWN'S QUEBEC HARBOR

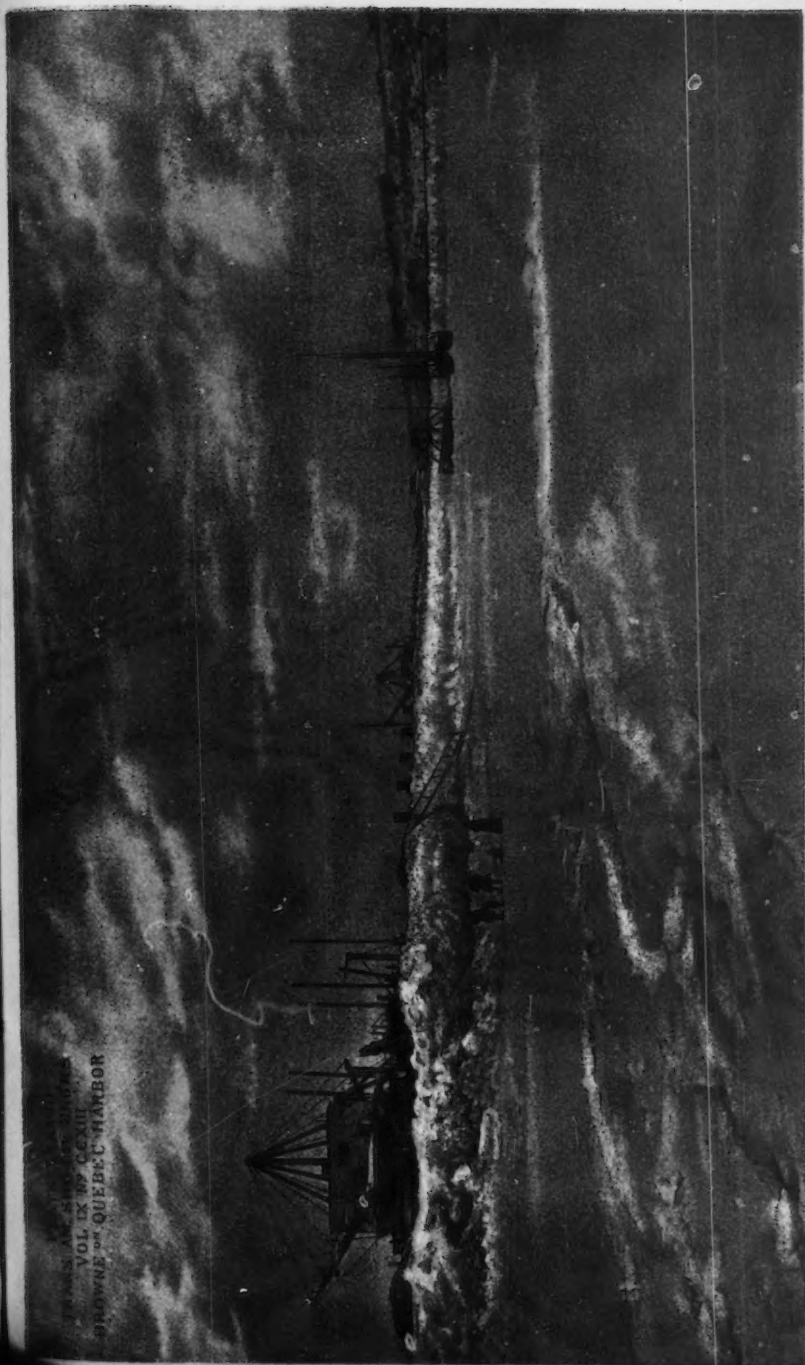




PLATE XXXIX.
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BROWNE ON QUEBEC HARBOR.

